Chapter 2
Static Soil Structure Interaction Problems

Soil structure interaction (SSI) problems are those where earth pressures depend on structure movements or deflections and structure movements or deflections depend on earth pressures. To analyze these problems, the foundation soil, the structure, and the backfill or retained soil must be considered. Examples of SSI problems include: anchored walls to stabilize landslides, cellular cofferdams, excavation bracing systems, long-span flexible culverts, reinforced slopes and embankments, retaining walls, U-frame locks, and tunnels.

2-1. Results and Use of SSI Analyses

a. *Principal results*. The principal results that can be obtained from an SSI analysis using finite elements are the stresses and displacements of the structure and the soil. In most real design problems, the stresses and displacements of the soil and structure can only be calculated using a numerical method like a finite element analysis. Conventional limit equilibrium methods, which do not predict displacements, are adequate for design where there is a sufficient base of experience. When there is less experience, or when displacements are critical, SSI analyses may be needed.

An SSI analysis can be used as a design tool in the following ways:

(1) Calculated values. Stresses and deformations of the structure and/or the soil can be calculated, and the calculated values can be compared to allowable values. If necessary, changes in the system configuration or the constructed component stiffnesses can be made, and the SSI analysis can be repeated until the calculated stresses and deformations are acceptable.

(2) Questions that arise. The "what if" questions that arise during the design process can be addressed in a rational manner. For example, due to subsurface heterogeneity and limited budgets for exploratory work and laboratory testing, significant uncertainty can exist in characterizing subsurface conditions. This uncertainty can create questions concerning the reliability of performance predictions. Such questions can often be addressed by performing parameter studies. Parameter studies on material property values are relatively easy to perform once an SSI model has been set up. If reasonable variations of material property values result in acceptable values of calculated stresses and deformations, further field and/or laboratory work may not be necessary. On the other hand, if reasonable variations of material property values result in unacceptable stresses or deformations, it may be necessary to modify the proposed construction in some way or to expend further effort to characterize subsurface conditions. In the latter case, the effort can be focused on those aspects of the problem that have been found by SSI analysis to be critical to performance.

b. *Efficient application*. In addition to its usefulness for predicting performance before construction, an SSI analysis using finite elements can contribute to efficient application of the observational method. The analysis results can be used to identify both representative and critical locations of installation of instrumentation that will be used to monitor performance during construction. Field measurements obtained from the instrumentation during early phases of construction can be used to calibrate the finite element model. The calibrated model can then be used to make more reliable predictions of final displacements and stresses and to evaluate whether specific contingency plans should be implemented.

2-2. Important Features of SSI Analysis

The following paragraphs describe several of the most important features and considerations for development of a good finite element model for an SSI analysis.

2-3. Material Behavior Models

a. *Linear elasticity*. Structural components in SSI analyses are most frequently modeled using linear elasticity. Rock units in the foundation are also frequently modeled using linear elasticity. Soil behavior, on the other hand, is usually more complex.

b. *Confining pressure*. As described in the introduction of this ETL, the stress-strain behavior of soil is nonlinear and inelastic. For all cases except saturated soil under undrained conditions, the stress-strain behavior of soil is dependent on confining pressure. These aspects of soil behavior are encountered in most geotechnical engineering projects, including projects where SSI is important. Consequently, it is important that the material model be capable of tracking these aspects of soil behavior.

c. *Material models*. Many material models, such as the hyperbolic model of Duncan and Chang (1970) and the Cam-Clay model (Roscoe and Burland 1968), do capture these characteristics of soils. The hyperbolic model uses a confining pressure-dependent, nonlinear elastic formulation, with an inelastic component introduced, because the value of the unload-reload modulus is larger than the value of the virgin loading modulus. The Cam-Clay model uses a plasticity formulation that also yields reduced modulus values as the soil strength becomes mobilized and increased modulus values as the confining pressure increases. One of the key benefits of plasticity is that it can model plastic strains that occur in directions other than the direction of the applied stress increment. This feature becomes especially important when a soil mass is near failure. In such a case, the application of a load increment in one direction can cause large displacements of the soil in another direction if large forces had been previously applied in that other direction. For well-designed structures in which failure of large masses of soil is not imminent, modeling this aspect of failure can become less important.

2-4. Stress-Strain Material Properties Values

a. *Material property values*. Selection of appropriate stress-strain material property values is often the most important step in performing SSI analyses. There are four methods to obtain material property values:

(1) Sampling and laboratory testing. For foundation soils, relatively undisturbed samples should be obtained. For embankment or backfill materials, laboratory compacted specimens can be prepared. In either case, the specimens should be tested in the laboratory in an appropriate manner to obtain the necessary parameter values for the material model that will be used. Typical laboratory tests for obtaining these values are one-dimensional (1-D) consolidation tests, isotropic consolidation tests, triaxial compression tests, and direct, simple shear tests.

(2) Field testing. Some *in situ* tests, e.g., the borehole pressuremeter tests, can be performed to obtain material property values.

(3) Correlations with index property values. Stress-strain material property values for several soils have been published together with index property values for the same soils, e.g., Duncan et al. (1980). These published values, together with judgment and experience, can be used to estimate appropriate stress-strain material property values based on index property test results for the soils of interest.

(4) Calibration studies. In many cases, designers have experience with local soils and are skilled at calculating 1-D consolidation settlements using conventional procedures. It is good practice in such cases to develop a 1-D column of finite elements that models the soil profile at the site of interest. The 1-D column can be loaded and the resulting settlements compared to those calculated using conventional procedures. The material property values for the finite element analyses can be adjusted until a match is obtained. Similarly, if an independent estimate of the lateral load response, i.e., the Poisson effect, can be made, the material property values can be adjusted until the 1-D column results match the independent estimate. Ideally, one set of material property values would be found that provides a match to both the compressibility and the lateral load response over the range of applied loads in the SSI problem to be analyzed.

b. *Selection of method*. The selection of a method to obtain material property values depends, of course, on the type of information available. The above methods are most effective when used in combination.

2-5. Finite Element Mesh

a. *Finite element mesh*. The finite element mesh for an SSI analysis should reflect the geometry of the structure, the stratigraphy in the foundation, and the configuration of any excavations and/or fills that are part of the work. In addition, the mesh should have sufficient refinement that deformations and stress gradients are smoothed as one moves from element to element in areas of interest.

b. *Known boundary condition*. The mesh should also extend beyond the area of interest until a known boundary condition is encountered (e.g., bedrock can often be represented as a fixed boundary condition) or for a sufficient distance that conditions at the boundary do not significantly influence the calculated stresses and deformations in the area of interest.

The finite element mesh for an SSI analysis can include several different types of elements:

(1) Two-dimensional (2-D) elements. 2-D elements for the soil and concrete portions of plane-strain and axisymmetric analyses.

(2) 3-D brick elements. 3-D brick elements for the soil and concrete portions of 3-D analyses (although it should be pointed out that 3-D analyses of geotechnical engineering problems are rare because of the great cost and time necessary for setting up the problem and interpreting the results, as well as due to the fact that many important aspects of 3-D problems can be modeled using 2-D meshes).

(3) Beam or shell elements. Beam or shell elements for sheet-pile walls, cellular cofferdams, and other structural components.

(4) Bar elements. Bar elements for struts and tiebacks.

(5) Interface elements. Interface elements to allow for slip between dissimilar materials such as between backfill soil and a concrete retaining wall.

2-6. Construction Sequence

a. *Construction sequence*. As described in the introduction, it is important to model the construction sequence in soil-structure interaction problems for two reasons: 1) soil response is nonlinear, and 2) the geometry can change during construction, e.g., fill placement.

b. *Initial in situ stresses*. Because of the nonlinear stress-strain behavior of soils, it is almost always necessary to first calculate the initial *in situ* stresses in the foundation materials. Perhaps the only exception occurs when a rock foundation is being modeled as linear elastic. In addition, it is necessary to model the following types of construction operations in steps: excavation, fill placement, placement and removal of structural components, and application of loads and pressures. Less often it can be important to model other sources of load, such as thermal strains in structural elements and compaction-induced lateral earth pressures, for example.

2-7. Calibration of the Entire Model

a. *Consider factors*. As can be seen from the foregoing, there are several factors that must be carefully considered to develop a good finite element model of the SSI problem. It is important to successful application of the method to calibrate the entire process against instrumented case histories. Fortunately, several such comparisons have been published (See Reference list in Chapter 5).

b. *View results with caution*. Whenever the method is applied in an unprecedented way, the results should be viewed with caution until confirmation by comparison with an instrumented case history can be established.

2-8. Case History: Retaining Wall at Bonneville Navigation Locks

a. *Project description*. A temporary tieback wall was built to retain the excavation for construction of a new navigation lock at Bonneville Dam on the Columbia River between Oregon and Washington. The geologic profile at the site, slide debris and man-made fill overlying rock units, is shown in Figure 2. The landslide occurred in the Pleistocene, and previous stability analyses had shown the landslide to be stable in its preconstruction configuration. Figure 2 also shows excavation and a railroad line relocation that took place prior to constructing the tieback wall. An important objective of the construction was to limit the magnitude of movements that would take place at the railroad line during excavation for the new navigation lock.

The 440-ft-long wall consists of a series of reinforced concrete panels. Each panel was excavated by rock chisel and clamshell, with the excavation supported by a bentonite-water slurry. After excavation of each panel, reinforcement was placed and the excavation was backfilled with concrete. The heights of the panels range from 20 to 110 ft. Following completion of the wall panels, excavation for the navigation lock

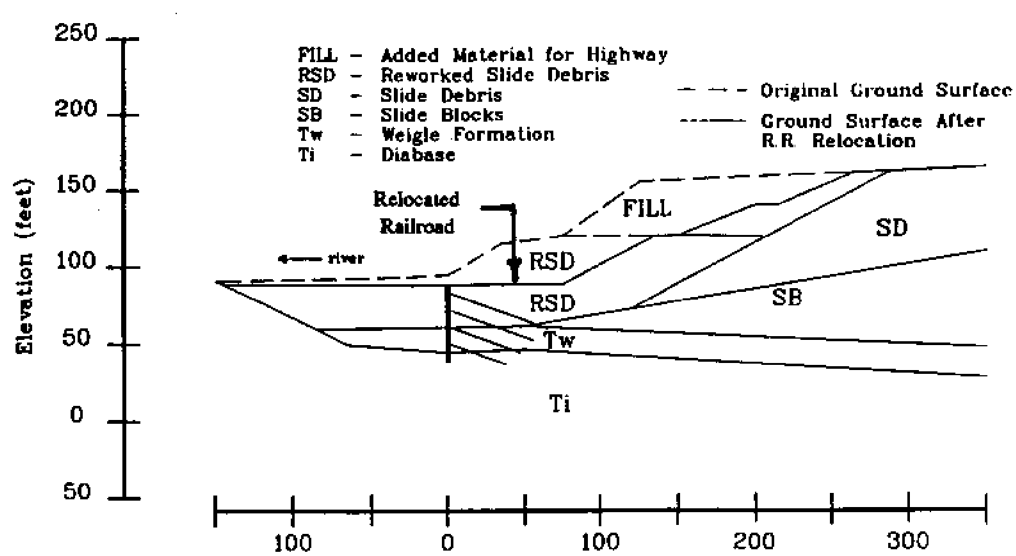

Figure 2. Geologic profile for analysis of Bonneville tieback wall

commenced, and tiebacks were installed at a grid spacing of approximately 10 ft by 10 ft.

b. Purposes of the SSI analysis. SSI analysis of the tieback wall were performed by Mosher and Knowles (1990) for three principal purposes:

(1) To confirm previous design studies based on limit equilibrium procedures and beam on elastic foundation analyses.

(2) To predict wall performance during excavation and tieback installation.

(3) To assist in the interpretation of instrumentation results.

c. Material behavior model and property values. The hyperbolic model (Duncan and Chang 1970) as implemented in SOILSTRUCT (Clough and Duncan 1969, and Ebeling et al. 1990) was selected for the soil and rock units at the site. Material property values were obtained from interpretation of laboratory test results. Structural materials were assumed to behave in a linear elastic manner.

d. Mesh details. The finite element mesh used for the analyses is shown in Figure 3. The mesh has 395 elements and 389 nodes, and it extends a considerable distance away from the wall. Two-dimensional elements were used to model the soil and rock units and the reinforced concrete wall. Bar elements were used to represent the tie-backs. Bar elements were also employed as "strain gages" on either side of the wall so that extreme fiber stresses and bending moments could be calculated in post-processing. Interface elements were used to allow slip between the wall and adjacent materials.

e. Construction sequence modeling. A series of analysis steps were used to develop the initial *in situ* stresses prior to construction. These steps included a gravity turn-on analysis followed by several steps to establish the initial ground surface slope. The displacements were set to zero at this point to represent the zero displacement condition prior to beginning construction.

The construction sequence was then modeled as follows:

(1) Excavate to the level of the top of the wall and the railroad grade.

(2) Excavate and place concrete for the retaining wall.

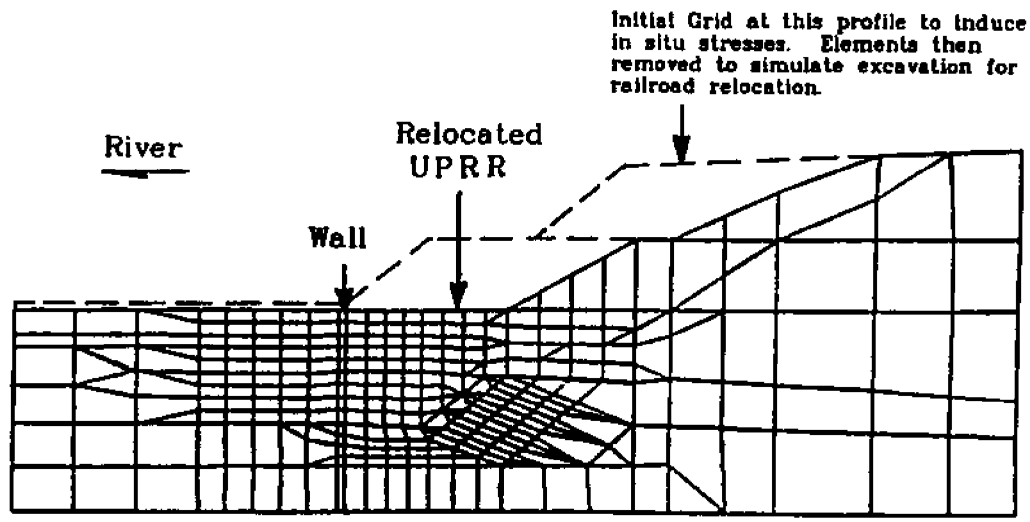

Figure 3. Finite element mesh for Bonneville tieback wall

(3) Excavate in front of the wall to the depth necessary for installing the uppermost tieback.

(4) Apply the tieback proof load.

(5) Reduce the tieback load to the lockoff load.

(6) Add the stiffness of the tieback to the mesh and excavate to the level of the next tieback.

Steps 4 through 6 were repeated until the bottom of the excavation was reached. These construction steps are illustrated in Figure 4 for four levels of tieback.

f. Results. The results of the analyses included values of wall deflections and moments, lateral earth pressures on the wall, ground surface movements behind the wall (including movement of the relocated railroad line), and soil stresses in the ground behind the wall.

Some of the results are shown in Figure 5. The calculated wall deflections at the end of the excavation are away from the excavation and toward the railroad line. This occurs because the large tierod reload forces pulled the wall toward the railroad line. The maximum calculated deformation is a small amount, 0.67 in. The calculated vertical movement of the ground surface at the railroad track alignment was 0.08 in. of heave. Figure 6 shows the calculated lateral earth pressure distribution on the wall at the end of construction in comparison to the lateral earth pressure distribution assumed during earlier design studies. The design earth pressures are larger than the calculated earth pressure at the top of the wall and smaller at the bottom of the wall. The undulations in the calculated pressure diagram result from the concentrated tieback reload forces that were applied.

g. Parameter studies during design. Parameter studies were performed during design to investigate the effects of reduction of the soil stiffness and the consequences of failure of the top anchor at the end of construction. The SSI analysis of the wall was repeated using values of soil stiffness equal to one-half of those obtained form laboratory tests. This change caused wall deflections to increase by about 65 percent, wall bending moments to increase by about 40 percent, and heave of the railroad line to increase by about 60 percent. The analysis of tierod failure resulted in a wall movement of about 1.45 in. toward the excavation, to a position 0.78 in. past the vertical. This lateral movement of the wall was accompanied by a 0.14-in. drop of the ground surface at the railroad line location, to a level of 0.06 in. below the original ground level.

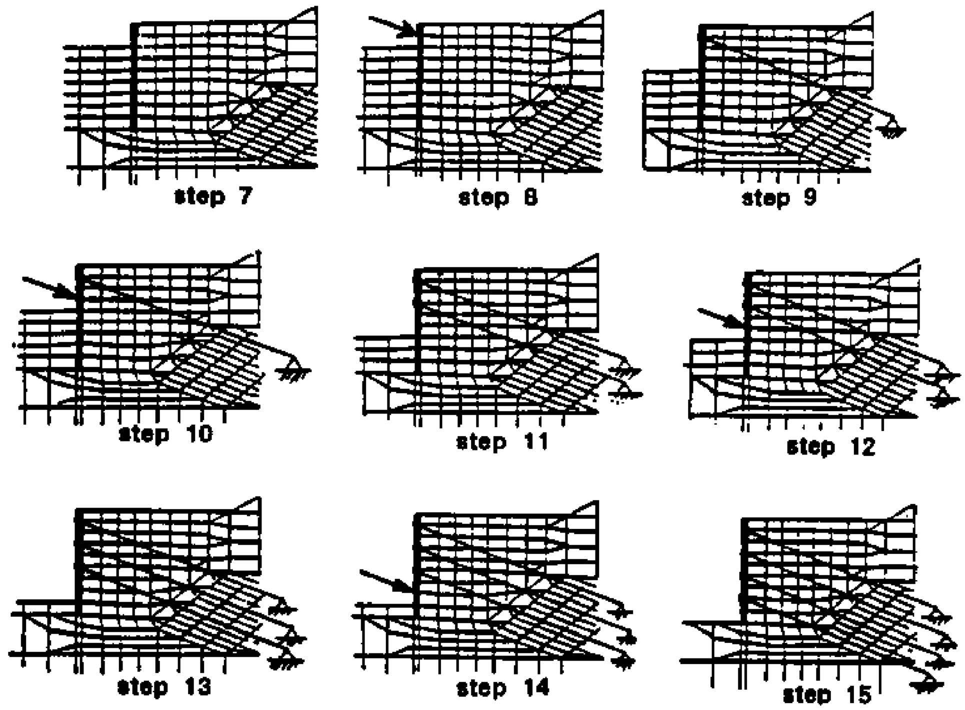

Figure 4. Finite element mesh for steps 7 through 15

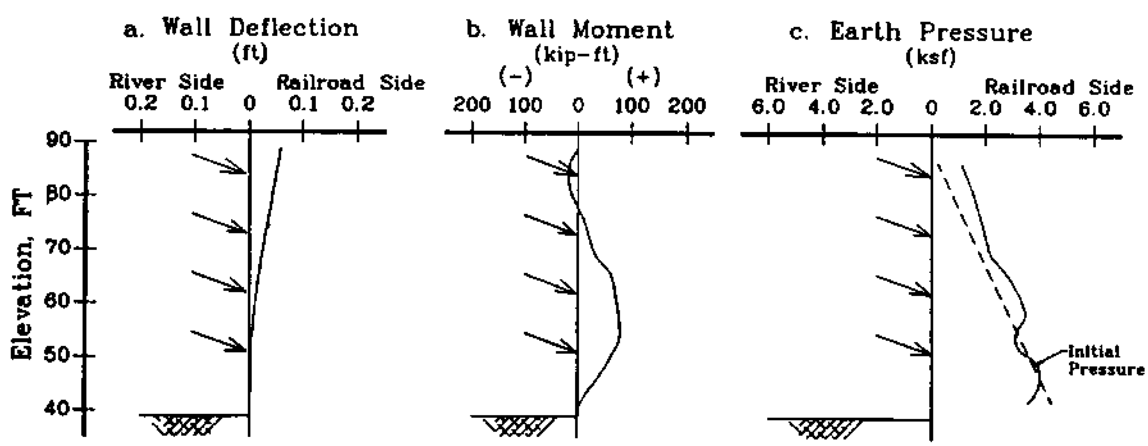

Figure 5. Wall deflections, moments, and lateral earth pressures after final excavation to elevation 39 ft with fourth anchor locked off

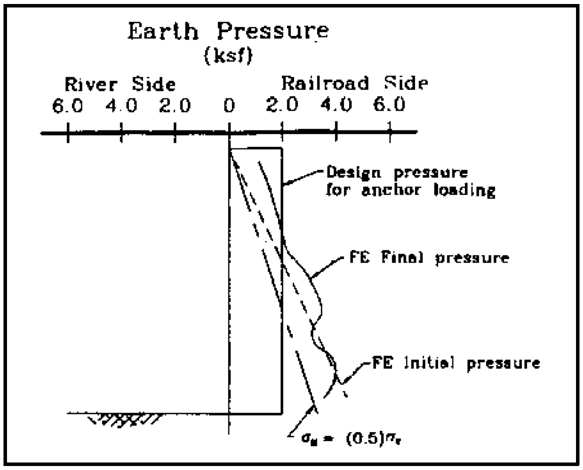

Figure 6. Earth pressures on tieback wall

h. *Comparison with measurements*. The wall was instrumented with inclinometer casings and strain gages. Figures 7 and 8 show the comparisons between calculated and measured deflections and bending moments. In both cases, the curve marked "Initial" represents the calculated values from the SSI analysis when the soil stiffness obtained from the laboratory tests was used in the analyses. The calculated deflections and moments exceeded the measured values.

i. *Parameter studies performed after making field measurements*. After the field measurements were obtained, additional parameter studies were performed in an attempt to better match observed behavior. By tripling the soil stiffness obtained from laboratory test data, a reasonably good match could be obtained. Figures 7 and 8 show the comparison. This result is in agreement with the experience on other projects that laboratory data frequently underestimate *in situ* soil stiffness.

The calibrated model could be used, if necessary, to calculate the response of the system to further loadings, such as surcharges or additional excavation.

2-9. Case History: Sheet-pile Wall Analysis

a. *Project description*. Sheet-pile walls are used for both flood protection along the Mississippi and Atchafalaya Rivers and hurricane protection along the Gulf of Mexico. The cost of the walls depends on the sheet-pile section and the depth of penetration required for stability. Conventional design practice incorporates both a limit equilibrium program and a beam-spring program to predict the stability of the system and the deflections of the sheet pile to determine if a given layout will meet design criteria. A full-scale test program and finite element analysis were performed as part of this study to investigate the effectiveness of the current design procedures.

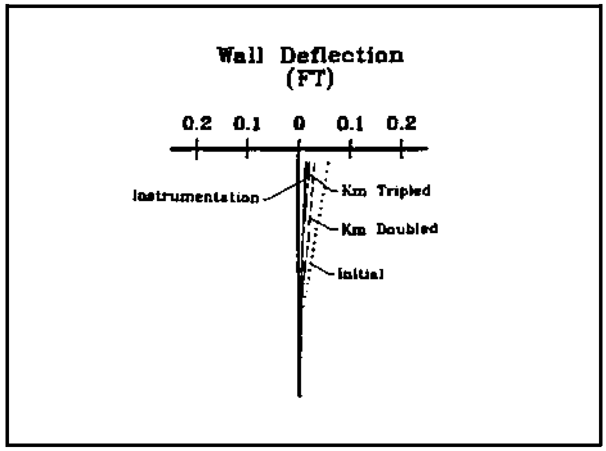

Figure 7. Calculated and measured wall deflections

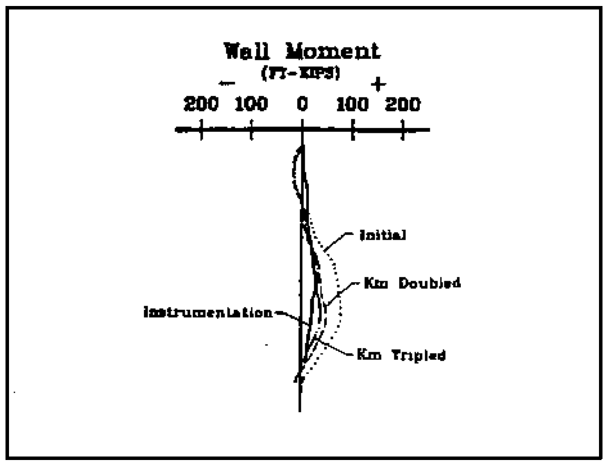

Figure 8. Calculated and measured bending moments

b. *Purposes*. This study had three primary puposes:

(1) To demonstrate the applicability of the finite element method to sheet-pile wall design in soft clays by analysis of the full-scale E-99 test section sheet-pile wall.

(2) To determine which factors have the greatest influence in the performance of the sheet-pile wall through a parametric study with the finite element method. Variations in soil properties, loadings, sheet-pile type, and depth of penetration were considered in this study.

(3) To develop recommendations for a sheet pile design procedure that overcomes some of the inconsistencies in the current methods.

c. Material behavior model, property values, and finite element code. The hyperbolic model (Duncan and Chang 1970) implemented in SOILSTRUCT (Clough and Duncan 1969, and Ebeling 1990) was selected for this problem. Soil material properties were determined from laboratory tests and back analysis of the observational data retrieved from the E-99 test section. The sheet piles were treated as linear elastic materials.

d. Modifications to finite element code. The finite element code, SOILSTRUCT, was modified during the course of the study to ease the input of material parameters for soils and to improve the means of computing the bending moments in the sheet-pile wall. These modifications included:

(1) Implementation of a (S_u/p) model to ease the input of shear strength parameters.

(2) Determination of the initial tangent modulus of soils, E_i, as a function of the undrained shear strength of the soil using the relationship

$$E_i = K \; x \; S_u \qquad \textbf{(1)}$$

where K is a unitless parameter between 250 and 1,000 as determined from previous experience.

(3) Improving the bending elements representing the sheet piles so that the bending moments could be directly computed.

e. Mesh details. The mesh used to model the E-99 test section is shown in Figure 9. The mesh consists of 281 solid elements and 322 nodes and models the foundation between elevations (el) +6.5 to -35 ft. Sheet-pile elements are attached to soil elements by 19 interface elements. Water loads are applied to the soil surface and pile as linearly varying distributed loads in increments corresponding to water levels of 4.0, 6.0, 7.0, 8.0, and 9.0 ft. A second mesh was used in this study for the purpose of performing a parametric analysis. This mesh, presented in Figure 10 and based on the E-105 test section, was used to investigate design implications of soft foundation behavior.

f. Construction sequence modeling. The basic construction/loading sequence employed in the finite element analyses of both the E-99 test section and in the parametric studies was:

(1) Computation of the initial stresses based on an elastic gravity turn on analysis.

(2) Insertion of the sheet pile.

(3) Application of water loads in 1-ft increments.

(4) Application of wave loads.

The stresses determined in (1) were used to determine S_u and the E_i for each element in the mesh. The insertion of the sheet-pile wall was accomplished by changing the material of the elements representing the sheet-pile wall from soil to steel during the first step. Water loads were simulated through the application of the appropriate pressure to surface nodes in contact with the floodwaters.

g. Results of the E-99 test section. Field data obtained from the E-99 test section was used to establish and validate the FEM for the analysis of the sheet-pile walls. A PZ-27 sheet pile was simulated in the analysis. Water loads were applied to simulate water levels of elevations 4.0, 6.0, 7.0, 8.0, and 9.0 ft. Soil material properties for analysis were obtained from "Q-tests" and field classifications. Three shear strength profiles obtained form test data, used in design, and used in the finite element analysis are shown in Figure 11. The soil stiffness in all finite element runs of the E-99 test section were made on the assumption that K was the same for all soils. Two runs were made with K = 500 and K = 1,000. Leavell et al. (1989) concluded from the SOILSTRUCT analysis that:

(1) Wall-versus-head relationship. The displacement at the top of the wall-versus-head relationship is predicted fairly well as shown in Figure 12. The ability of the analysis to predict the larger displacements as the head approached 8.0 ft is particularly important because it implies that the limit

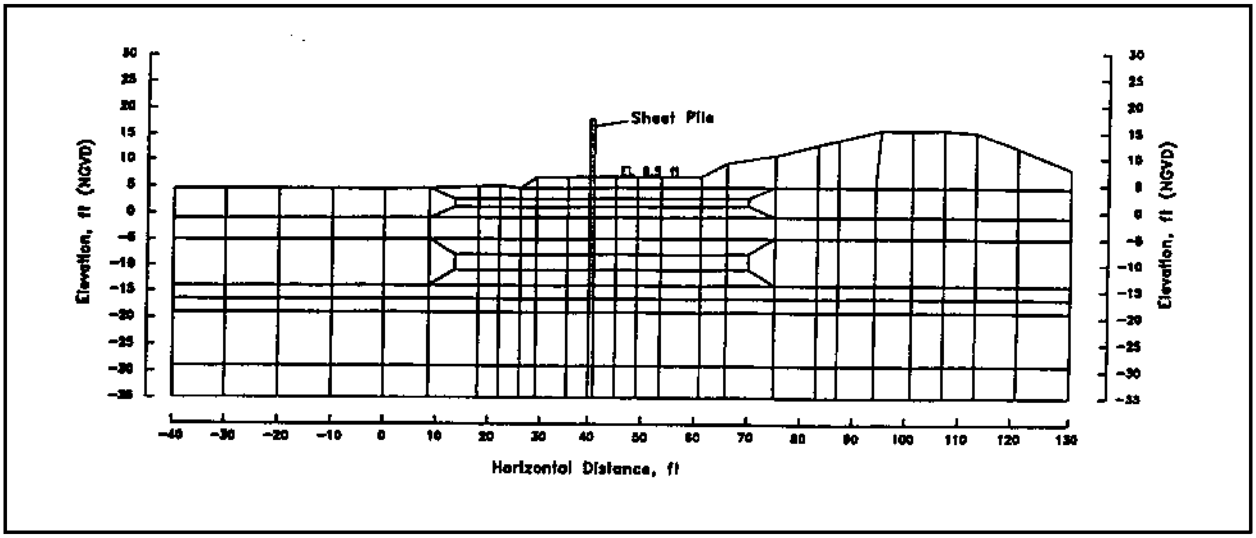

Figure 9. Finite element mesh for E-99 test section

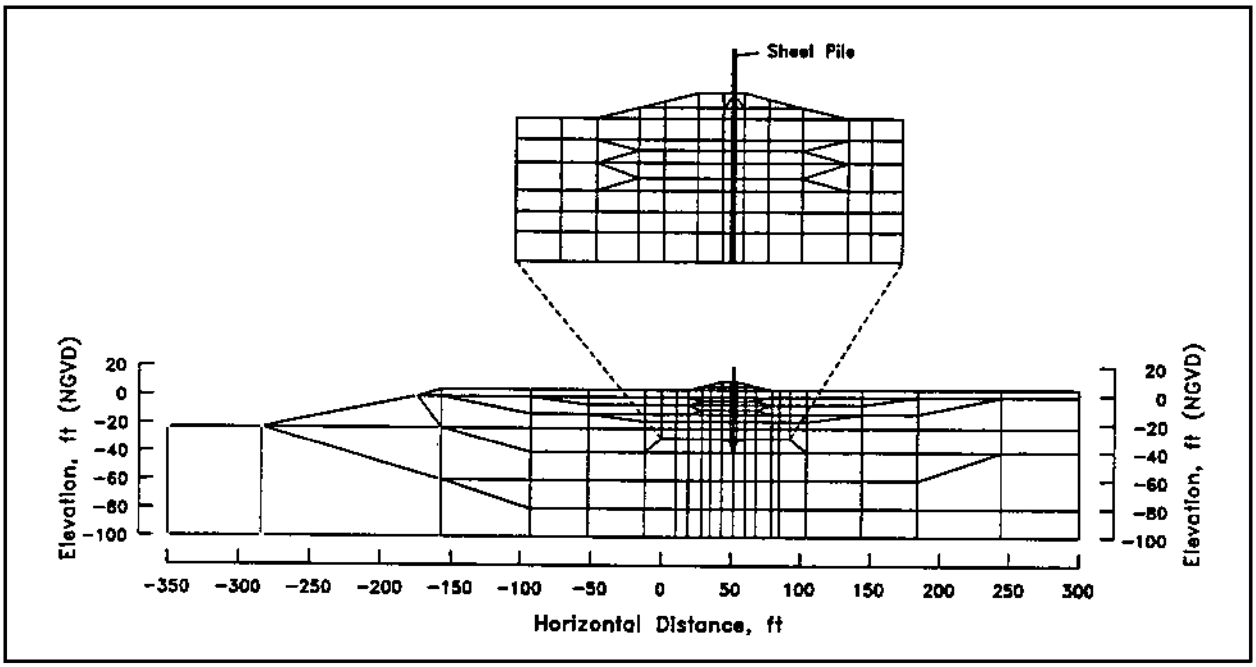

Figure 10. Finite element mesh for E-105 levee section

load can be computed accurately with the finite element method.

(2) Distribution along the wall. The displacement distribution along the wall is predicted well as shown in Figure 13. The ability to predict displacements near the pile tip is significant because in the soft-soil foundation deep-seated movements can control the displacements of the pile-levee system.

(3) Computed maximum moments. The computed maximum moments and their location agreed well with those measured in the field as shown in Figure 14.

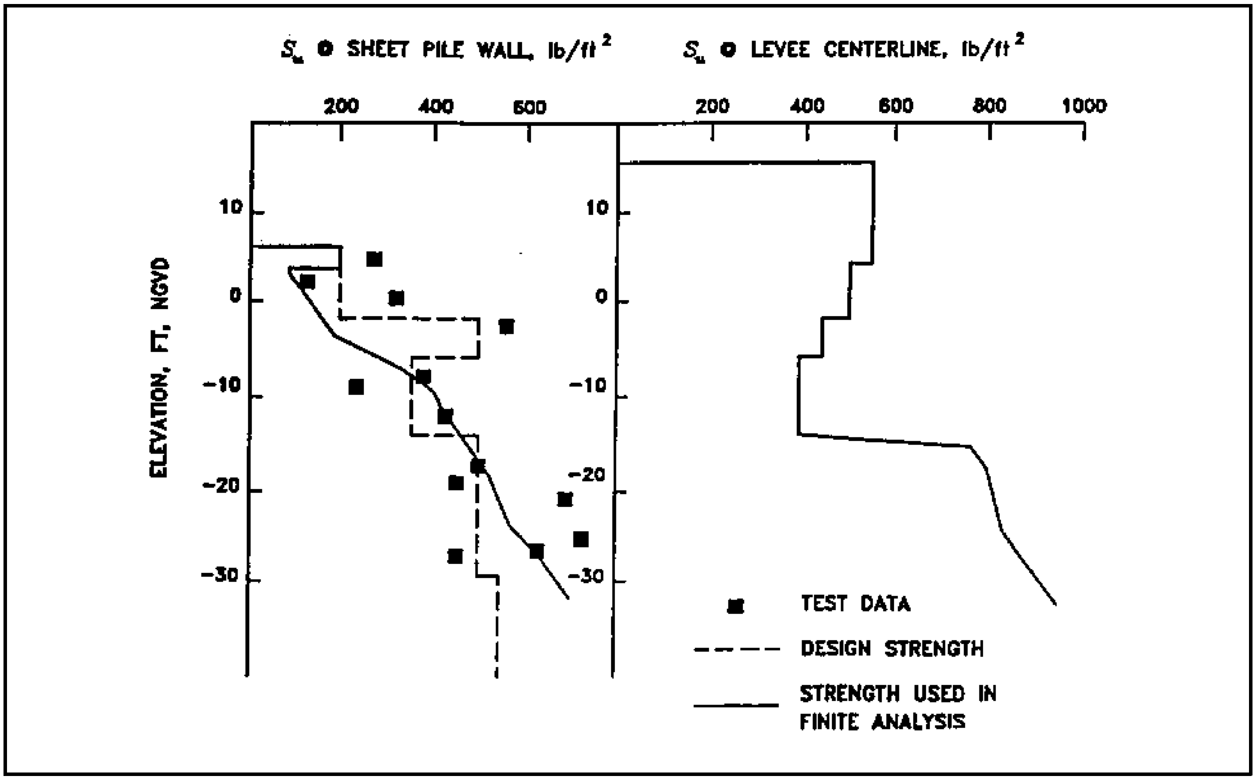

Figure 11. Undrained shear strength profile for Section E-99

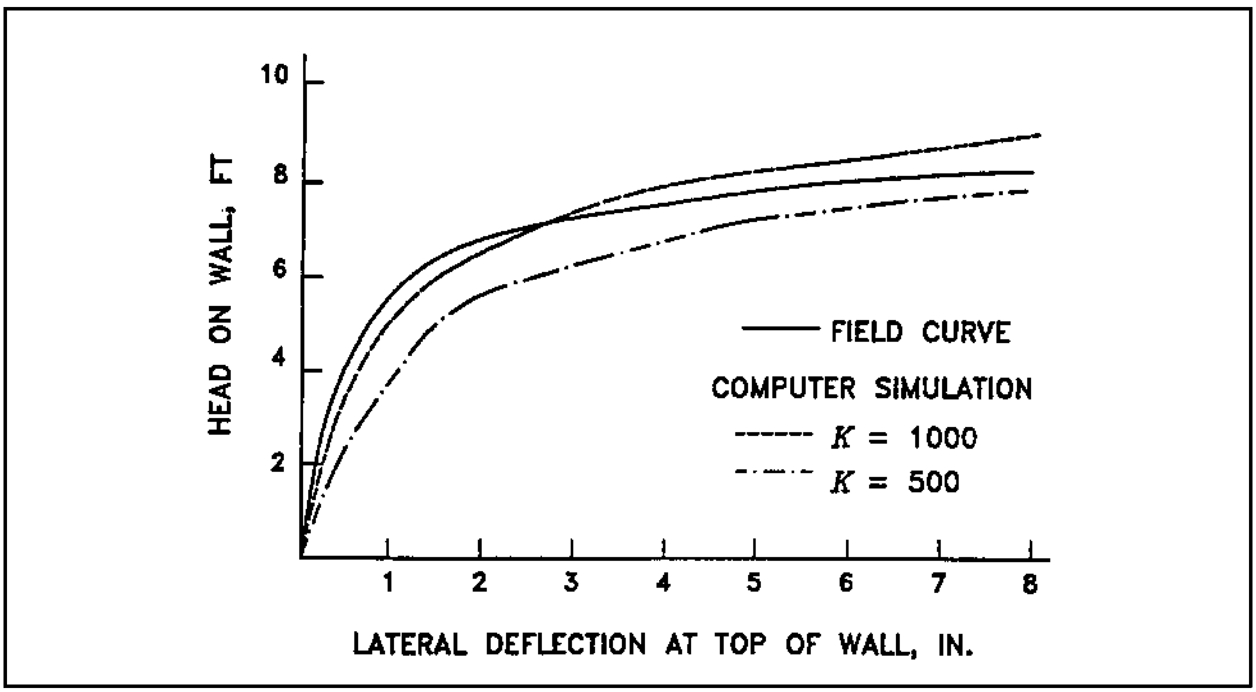

Figure 12. Computed and measured deflections at top of wall versus head for section E-99

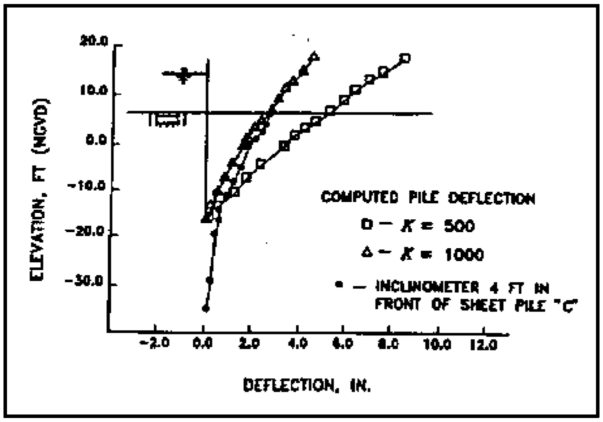

Figure 13. Computed and measured deflections of sheet-pile wall

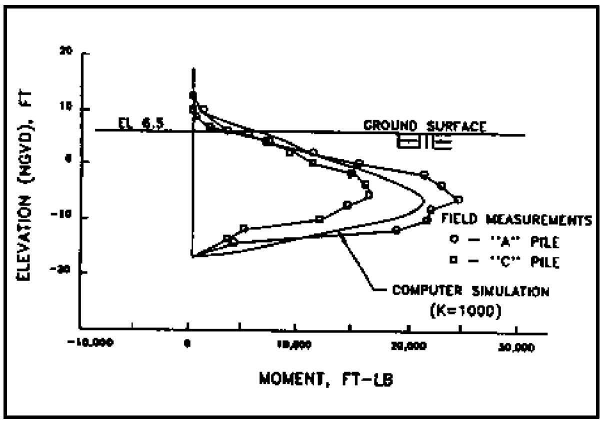

Figure 14. Computed and measured bending moments in sheet-pile wall

h. *Results of the parametric analysis*. The parameter study was designed to evaluate the effects of pile embedment depth, soil strength, and pile type on the performance of the system at various water levels. The finite element analyses were performed in conjunction with a limit equilibrium analysis to establish a link between the displacements computed with the FEM and the safety factor computed with the limit equilibrium method. Some of the key findings of the parametric analysis include:

(1) Deep-seated movements. Deep-seated movements in the levee foundation controlled the magnitude of sheet-pile deflection, particularly in soft soils. As a result, the height of water loading that can be sustained by a particular wall is controlled by the stability of the foundation, as determined by a slope stability analysis.

(2) Stability of the levee. The stability of the levee implied by the displacements is consistent with the safety factor computed by the limit-equilibrium method. This is shown in Figure 15 where the sheet-pile wall movements are sensitive to safety factors less than about 1.3.

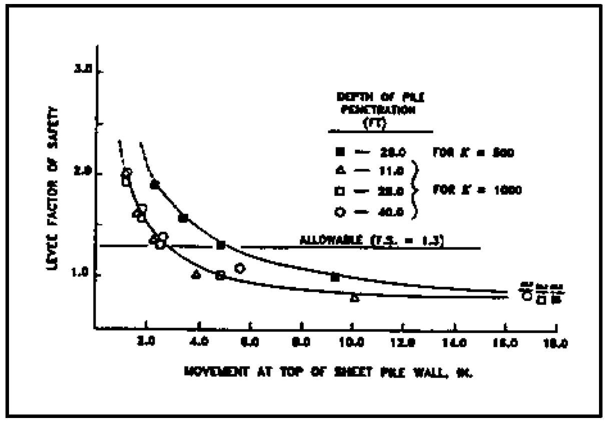

Figure 15. Displacement computed by the finite element method versus factor of safety computed by limit equilibrium method

(3) Increased pile penetration. Increased pile penetration does not improve the stability of the levee.

(4) Pile stiffness. Pile stiffness has little effect on the total displacements.

(5) Deflection of sheet-pile wall. Deflection of the sheet-pile wall, as determined with conventional design programs, is a poor criterion for design of sheet-pile walls because movements are caused by shear deformations in the foundation and not the cantilever action of the pile.

Based on the findings of the parametric analysis, Leavell et al. (1989) were able to successfully develop a design procedure based on the finite element for sheet-pile design. The procedure gives designers charts for making a “correction” to the displacements computed with the conventional design programs. This correction accounts for the effect of the deep-seated movements on the pile deflections.